VOLUME 80 SEPARATE No. 568

PROCEEDINGS

AMERICAN SOCIETY OF CIVIL ENGINEERS

DECEMBER, 1954

DISCUSSION OF PROCEEDINGS - SEPARATES

432, 438, 439, 489

HYDRAULICS DIVISION

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Printed in the United States of America

Headquarters of the Society
33 W. 39th St.
New York 18, N. Y.

PRICE $0.50 PER COPY

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This paper was published at 1745 S. State Street, Ann Arbor, Mich., by the American Society of Civil Engineers. Editorial and General Offices are at 33 West Thirty-Ninth Street, New York 18, N.Y.

Discussion of "MORNING-GLORY SHAFT SPILLWAYS: DETERMINATION OF PRESSURE-CONTROLLED PROFILES"

by W. E. Wagner
(Proc. Sep. 432)

FRED W. BLAISDELL,[1] M. ASCE.—Mr. Wagner has interestingly presented the results of his experiments on the determination of pressure-controlled profiles. A by-product of this investigation—and a valuable by-product—is the data obtained on the discharge coefficient. The writer's comments will be largely on this phase of the paper.

A great many spillways, especially in the smaller sizes, do not have a shaped crest. Many of them are formed of pipe and the entrance takes the shape of the pipe end. The crest shape for metal pipe is close to the shape of the sharp-edged weir used by Mr. Wagner. Therefore, the results obtained by him can be applied directly to the design of these spillways. A photograph of the type of structure which the writer has in mind was presented in a paper written by L. Standish Hall,[2] M. ASCE.

When discussing the discharge coefficient under the section, "Interpretation of Results," Mr. Wagner makes the statement, ". . . most morning-glory spillways are designed for free flow." The data presented by J. N. Bradley,[3] A. M. ASCE, show that the Hungry Horse, Gibson, and Owyhee Dams have values of H_S/R varying from a little above to a little below 0.45, while the Shade Hill, Heart Butte, and Kingsley Dams have values of H_S/R greater than 1. Mr. Wagner, under "Analysis of Results," states that free flow exists for values of H_S/R less than 0.45 and submerged flow exists for values of H_S/R greater than 1.00. Therefore, it appears that a sufficient number of dams operate in the submerged flow range (three of four dams completed in the United States since 1940) to make the statement open to question.

Since the orifice discharge coefficient $C_O = (2/\sqrt{2g})\ (H_S/R)\ C$, and since Mr. Wagner already has values of H_S/R and C, the writer would like to suggest the computation and presentation of values of C_O as part of the closing discussion. (Scaling from the base curve in Figure 9, the writer obtains C_O = constant = 0.51, and $C_O\sqrt{2g} = 4.1$ for values of H_S/R above 1.0.) This would be a valuable addition to the paper, would permit the use of the coefficients in the orifice equation (which is theoretically correct for values of H_S/R exceeding 1.00), and would permit extrapolating beyond the range covered by Mr. Wagner's tests.

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1. Project Superv., Agri. Research Service, U. S. Dept. of Agriculture, St. Anthony Falls Hydr. Lab., Minneapolis, Minn.
2. "Drop Structures for Erosion Control," by L. Standish Hall, Civil Engineering, Vol. 12, No. 5, May, 1942, pp. 247-250.
3. "Morning-Glory Shaft Spillways: Prototype Behavior," by J. N. Bradley, Proceedings, American Society of Civil Engineers, Vol. 80, Separate No. 431, April, 1954.

It is unfortunate that no data were obtained for a zero depth of approach (P/R = 0). This is a condition that exists on many smaller structures where the vertical shaft or drop inlet is formed of pipe and the approach is level with the crest.

Tables 4 and 5 provide interesting verification of Mr. Wagner's statement, apparently derived from a study of Figures 8 and 9, that the weir becomes partly submerged at values of H_s/R above 0.45 (see section headed, "Analysis of Results"). It is usually considered that the tailwater level will not affect the head on a weir unless the tailwater reaches a level above the weir crest exceeding the critical depth of flow d_c. Since $d_c = (2/3) H_s$, the value of Y/H_s when Y is equal to the critical depth is 0.667. Tables 4 and 5 show that the value of H_s/R for $Y/H_s = 0.667$ at the point where the upper nappe surface joins the boil is between 0.45 and 0.50. The value of H_s/R may be determined more precisely by plotting values of Y/H_s against H_s/R, which shows that $H_s/R = 0.47$ for $P/R = 2.00$, and 0.49 for $P/R = 0.30$ and 0.15, when $Y/H_s = 0.667$. These values closely confirm Mr. Wagner's value of 0.45.

For the free-flow portion of the base curve in Figure 9, Mr. Wagner shows a variable discharge coefficient. Camp and Howe,[4] M. ASCE, state that C is a constant equal to 3.28 within one per cent for values of H_s/R between 0.16 and 0.40. Unfortunately, Camp and Howe do not give values of C_o so these comparisons are not possible.

I. M. MOSTAFA ELASSIOUTY[5] J.M. ASCE - B. CHANDA[6] - HOWARD P. JOHNSON.[7]—Mr. Wagner has presented valuable experimental results that are of direct use in the design of morning-glory spillways. Some interesting analytical and experimental comparisons with these results can be made. Although somewhat different geometry is used in part of the analysis, the comparisons support the data presented by the author.

Dimensional analysis can be employed to account for all of the physical variables that affect flow over circular weirs. The variables pertinent to this case can be expressed in the form

$$Q = \phi(R , P , H_s , h_p , g) \qquad (1)$$

and

$$Y = \phi(R , P , H_s , h_p , X) \qquad (2)$$

in which the nomenclature is the same used by the author. Equations (1) and (2) can be put into the dimensionless forms

$$C_d = \frac{Q}{R\, H_s^{3/2} \sqrt{g}} = \phi\left(\frac{P}{H_s} , \frac{H_s}{R} , \frac{h_p}{H_s}\right) \qquad (3)$$

and

4. "Tests of Circular Weirs," by Cecil S. Camp and J. W. Howe, Civil Engineering, Vol. 9, No. 4, April, 1939, pp. 247-248.

5 6 7. Respectively Post-Doctoral Fellow; and graduate students, State University of Iowa, Iowa City, Iowa.

$$\frac{Y}{H_s} = \phi\left(\frac{p}{H_s}, \frac{H_s}{R}, \frac{h_p}{H_s}, \frac{X}{H_s}\right) \qquad (4)$$

The coefficient of discharge C_d and the nappe profile will depend upon the geometry of the weir and the pressure intensity below the nappe. The functional relationship indicated in Eqs. (3) and (4) can be determined experimentally.

If the weir is completely submerged and if the depth of approach is infinite, a relationship for the coefficient of discharge can be derived analytically for the two-dimensional counterpart of the flow. Although this analytical approach has been developed for two-dimensional flow only, the results obtained can be applied with good accuracy to the three-dimensional flow[8,9] - and a comparison between the results obtained by the author in Fig. 9 for the range in which the weir is submerged and the velocity of approach is negligible is relevant to the author's purpose in presenting the study.

Figure 1a is a definition sketch of the comparable two-dimensional case. Revolution of the boundary BL around DC would give the boundary of a circular weir. The free water surface above the weir, which is practically level when the flow is submerged, is represented by ED. The weir becomes an orifice when it is submerged and the gravitational effect on the coefficient of contraction is known to be definitely secondary. For this reason an ultimate width of the nappe can be assumed. By means of a series of mathematical transformations, the flow is transformed into a much simpler pattern. The three transformations used in the following analysis are shown in Fig. 1. For simplicity of nomenclature the ultimate width of the nappe and the velocity of the free streamline have been taken as unity.

The flow is represented in the logarithmic hodograph (the Ω plane) by the usual relationship

$$\Omega = -\ln V + i\,\theta$$

for which the flow region takes the form indicated in Fig. 1b. This latter region is in turn mapped onto the upper half of the auxiliary t-plane. As indicated in Fig. 1c, the value -1,0 and 1 are assigned to the points C, E, and B in the t-plane. The value of t for D and A can be denoted by -f and f respectively.

The appropriate relationship between Ω and t is then obtained by the use of the Schwarz-Christoffel transformation

$$\Omega = -i \tan^{-1}\left(\sqrt{\frac{t^2 - 1}{1 - F^2}}\right) + i\frac{\pi}{2}$$

The constants in the transformation were evaluated by substitution of the values of Ω and t for points E and F in the two planes.

The complex potential ω can be expressed in terms of t by the methods of elementary hydrodynamics:

8. "Characteristics of Irrotational Flow Through Axially Symmetric Orifices", by H. Rouse and A. Abul-Fetouh, Trans. ASME Vol. 17, 1950, p. 421.
9. "Free-Streamline Analyses of Transition Flow and Jet Deflection", edited by John S. McNown and Chia-Shun Yih, State University of Iowa, Studies in Engineering, Bulle. 35, Published in 1953.

$$\omega = \frac{1}{\pi} (\ln (t + 1) - \ln (t - f))$$

Completion of the analyses consists of the algebraic definitions of the lower nappe as a function of the assigned value f. The relationship between the horizontal coordinate of the outer nappe surface (b) and the head above the weir crest (H_s) may be expressed in two equations as follows:

$$b - 1 = \frac{1}{\pi} \int_1^{-1} \left(\frac{1}{t - f} - \frac{1}{t + 1}\right) \sqrt{\frac{t^2 - 1}{t^2 - f^2}}\, dt \qquad (5)$$

and

$$H_s = \frac{\sqrt{1 - f^2}}{\pi} \int_1^{-f} \left(\frac{1}{t - f} - \frac{1}{t + 1}\right) \frac{dt}{\sqrt{t^2 - f^2}} \qquad (6)$$

For any given value of f, the two equations can be solved for b and H_s. The integration in equation 5 can only be made numerically. In Eq. 6 the integrals are elementary and have been taken from tables; the result is

$$H_s = \frac{1}{\pi} \left(1 + \frac{1}{f} + \ln. \frac{1}{f}\right)$$

For the two-dimensional case the coefficient of discharge C_d, provided there is no loss of energy, is simply $\frac{1}{b}$.

The computed results are compared with the author's observed results on the basis of the assumption that the ratio $\frac{H_s}{b}$ and $\frac{H_s}{R}$ are identical. Values of $C = C_d \frac{\sqrt{2g}}{2} \frac{b}{H_s}$, the form used by the author, against $\frac{H_s}{b}$ have been plotted in Fig. 2, in which results from the author's Fig. 9 are included. The coefficient of discharge computed analytically correspond remarkably well to the experimental coefficients for $\frac{H_s}{R} > 0.8$. For $\frac{H_s}{R} \geq 0.8$ the surface above the weir is practically horizontal, which is the condition assumed for the upper free surface in the analytical solution. The values of the coefficients of discharge C obtained from the test results are consistantly slightly higher than those computed for any value of $\frac{H_s}{R}$. This may be partially explained by the fact that the latter were obtained for an infinite depth of approach.

The profile of aerated nappe surfaces presented by the author in his Fig. 8 are almost exact reproductions of the profiles presented by C. S. Camp.[10] Figure 3 shows the position of points selected from Camp's data and plotted

10. "Determination of Shape of Nappe and Coefficients of Discharge of a Vertical Sharp-Crested Weir, Circular in Plan, with Radially Inward Flow," by C. S. Camp, a thesis submitted in the State University of Iowa in 1937 for the degree of Master of Science.

on the profiles determined by Mr. Wagner. The earlier work was conducted using weir arcs with radii 3, 2, and 1 foot having circular angles of 29, 44, and 97 degrees, respectively.

Regarding the effect of vacuum on the nappe profile, Fig. 4 reveals that as the submergence increases the effect of vacuum on nappe displacement diminishes. In Fig. 4 the lower nappe profiles for $\frac{H_s}{R} = O$ were secured from an investigation made by L. A. Thorssen.[11] The 10% increase in vacuum, defined as the ratio of observed pressure under the nappe in feet of water to total head above the weir crest, shifts the profile considerably for $\frac{H_s}{R} = O$, however for $\frac{H_s}{R} = 0.80$ only a slight movement of the under side of the nappe is observed for changes of vacuum from 10% to 20%. As the spillway becomes completely submerged vacuum apparently acts as an increase in head and does not materially change the shape of the under side of the nappe.

It has been shown that the results presented by the author, which are well supported by earlier investigations of a similar nature, are in close agreement with the analyses presented herein for the case of a two-dimensional submerged weir when the depth of approach is infinite.

11. "Effect of Vacuum on a Free Nappe", by L. A. Thorssen, a thesis submitted in the State University of Iowa in 1946 for the degree of Master of Science.

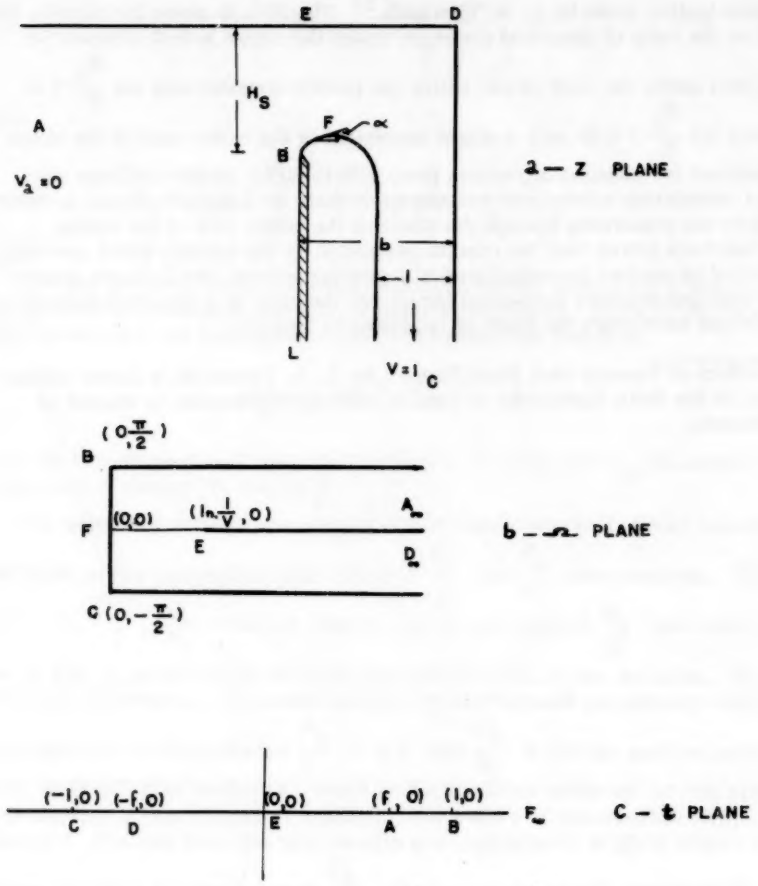

FIGURE 1. TRANSFORMATION PLANES

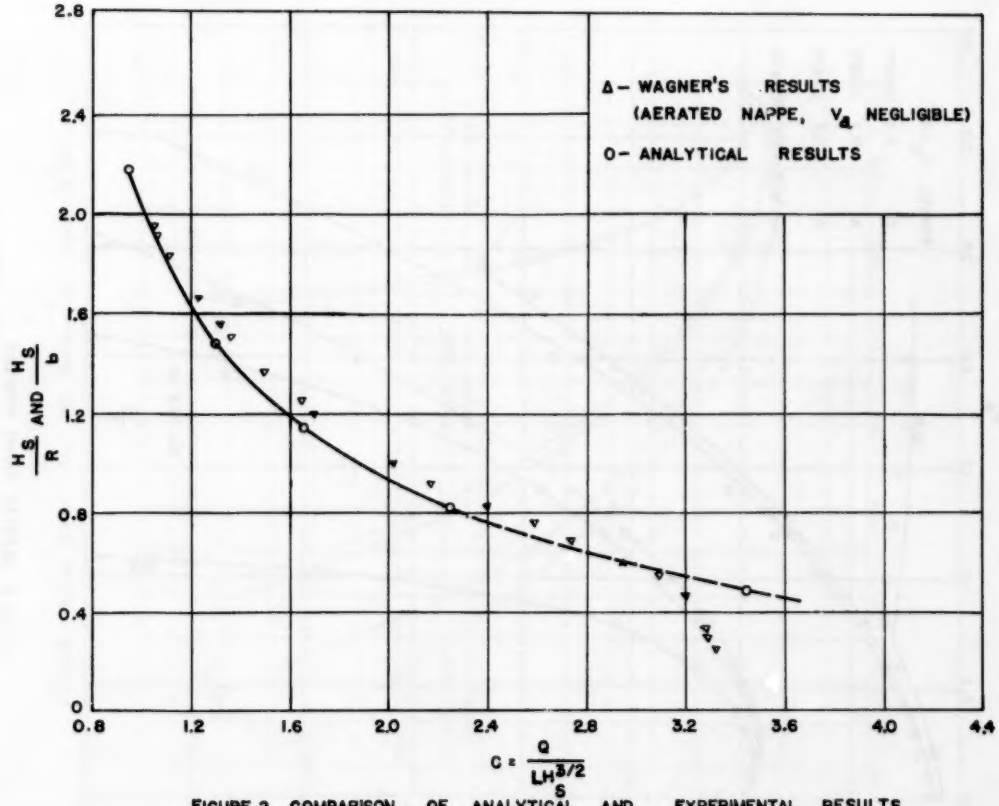

FIGURE 2. COMPARISON OF ANALYTICAL AND EXPERIMENTAL RESULTS

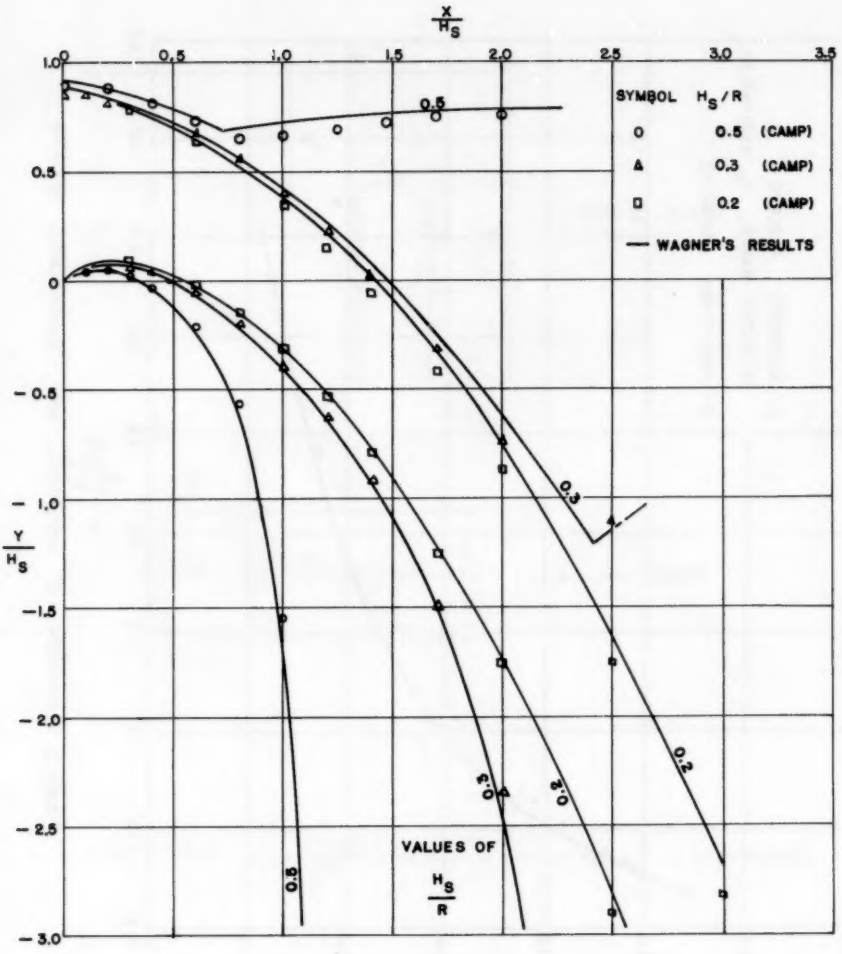

FIGURE 3. AERATED NAPPE PROFILES
(NEGLIGIBLE VELOCITY OF APPROACH)

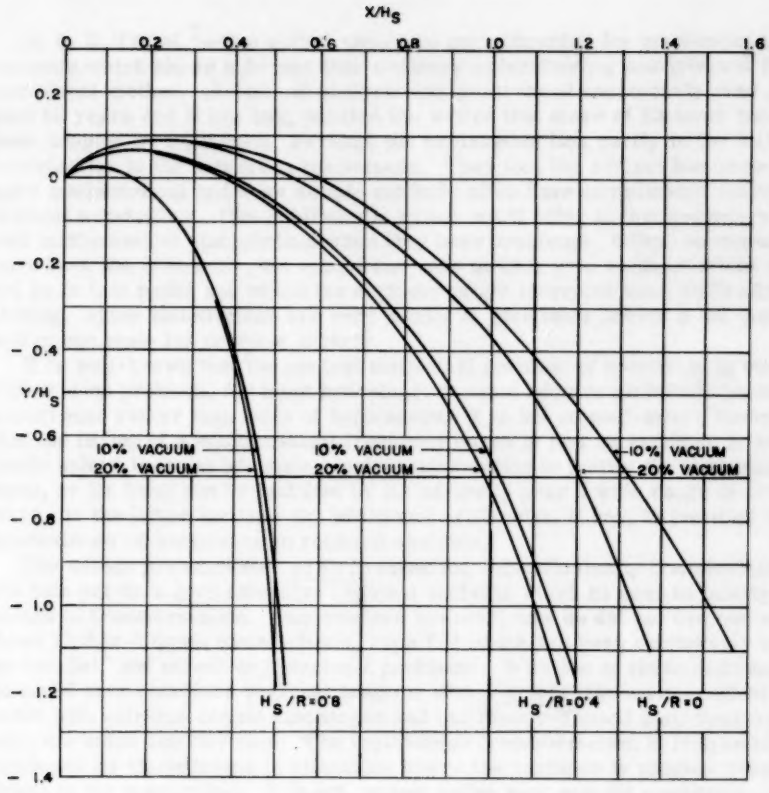

FIGURE 4. EFFECT OF VACUUM ON THE LOWER NAPPE PROFILE

Discussion of "ESTIMATION OF FLOOD PROBABILITIES"

by Leo R. Beard, A.M. ASCE
(Proc. Sep. 438)

H. C. S. THOM.[1]—The author should be complimented for presenting some analysis which shows a deeper than ordinary understanding and interest in the statistical method. Statistical analysis has progressed enormously over the past 50 years and it has long puzzled the writer that more of its tools have not been adopted by engineers. Perhaps the explanation lies partly in the lack of introduction to the subject in the schools. Then too, the subject has become very mathematical and even simple methods often have complicated mathematical foundations. One solution the writer could offer is that engineers consult mathematical statisticians when they have problems. Often, as would have been the case here, the expert can very quickly give methods which may not be in text books and which the engineer might therefore have difficulty finding. Many statisticians are very willing to give their advice if the consulter can state his problem clearly.

It is well-known that the central statistical problem of hydrology is the distribution problem, for most hydrologic designs require probabilities of occurrence rather than tests of hypotheses. It is not so well-known however that the fitting of a mathematical frequency curve is just an exercise in arithmetic unless there is adequate a priori information to justify its mathematical form, or its form can be justified by its adequacy over a wide range of conditions. In the latter instance the additional arithmetic, if any, is justified by convenience or assistance in regional analysis.

The author presents no a priori reason for logarithmically transforming his data but does give extensive regional analysis which he uses to justify his choice of transformation. One wonders however, why he did not use one of the three Fisher-Tippett distributions, Type I of which has been extensively used by Gumbel[1] and others in hydrologic problems. With one of these distributions he could have combined regional analysis with a priori information, since he deals with extreme annual discharges and the Fisher-Tippett distributions are extreme value distributions. The logarithmic transformation is frequently employed by statisticians in situations where the variance is roughly proportional to the mean value. It is not, except under very special conditions, an exact transformation to normality and so is rarely used where the objective is to estimate probabilities.

The author's procedure is equivalent to having chosen one type of logarithmic-normal distribution (LND) which as the writer[3] showed is its simplest form. It was also shown there why more general forms often fit

1. Chief Climatologist, Weather Bureau, U. S. Dept. of Commerce, Washington 25, D. C.
2. Probability Tables for the Analysis of Extreme-Value Data, Natl, Bur. of Stand., Applied Math. Series 22, 1953.
3. The Generalized Log-Normal Distribution Applied to Stream Discharge, A.G.U. meeting May 4, 1953, Washington, D. C.

stream discharge data better, but since the author made no tests of fit there was no convenient method of comparison. The author's justification for use of the normal distribution as the "theoretical distribution of flood flows" might apply to the original discharges (although at best these classical justifications are poor substitutes for a priori information and are now rarely referred to), but it does not apply to the logarithms for any similarity to normality explanation after logarithmic transformation must be purely accidental. The justification for logarithmic transformation for certain types of data was given by Seeliger.[4]

The author has done a very interesting analysis with the skewness and flatness statistics. The extremely low correlations are remarkable and certainly indicate a lack of persistence of either characteristic. He could have applied more exact tests of significance combined with his methods although it is doubtful whether he would have obtained essentially different results. (See the writer's paper.)[5]

The author has estimated confidence bands by a procedure which does not appear to produce valid results. He obtains these from a combination of errors determined from the χ^2 - and t-distributions. This procedure will overestimate the width of the confidence interval because the addition of two 0.05 values will produce a value which has a probability less than 0.02.

The correct theory has been known at least since 1951[6] and is based on the fact that for normal distributions the author's deviate x is distributed not as t or χ^2 but as non-central t. The non-central t-distribution was not developed by "Student". It is the distribution of a similar variate but different in that for the t-distribution the mean is zero where as for non-central t the mean is not zero. The mean of a normal deviate is not in general zero; hence, the non-central character of the distribution required.

The writer would ordinarily use the non-central t-tables, but since these are not available to engineers a very close approximation due to Jennett and Welch[7] will be employed. In the author's notation $x = (L - M)/s$ (s replaced by $\bar{s}$) x is distributed as non-central t. The inequality $x > x_p$, where p is the probability of exceeding the confidence limit x_p, is equivalent to the equality $(M + x_p s) < L$. Our interest is then in the probability $P(M + x_p s < L\ U) = p$ where U is any fixed value of x determined by the estimates M and s from normal tables. M is distributed normally about μ with variance σ^2/n where σ^2 is the population value of s^2. The mean of s is a s and its variance is $b^2\sigma^2/2(n-1)$ where a and b are correction factors tabled by Jennett and Welch. (For large n, a and b tend to one.) Since M and s are uncorrelated, the variances add and hence $(M - x_p s)$ is nearly normally distributed with variance

$$v(M - x_p s) = \left(\frac{1}{n} + \frac{b^2 x_p^2}{2(n-1)}\right)$$

and mean $(\mu + x_p a \sigma)$. Further development then gives the quadratic equation

4. Astron. Nachrichten. vol. 132 (1893) col. 209.
5. Seasonal Degree-day Statistics for the United States, Mon. Weath. Rev. vol. 80 (1952) pp. 143-147.
6. R. A. Fisher, The Sampling Error of Estimated Deviates, British AAS, Math. Tables, vol 1.
7. The Control of Proportion Defective etc., Suppl. Jour. Roy. Stat. Soc. v. II, No. 1, 1939, p. 80.

$$x_p^2\left[a^2 - \frac{2.71\,b^2}{2(n-1)}\right] - 2aUx_p + \left[U^2 - \frac{2.71}{n}\right] = 0$$

whose roots are the confidence limits $x_{.05}$ and $x_{.95}$. For n = 20 of the author's examples this simplifies to

$$0.902\,x_p^2 - 1.974\,Ux_p + U^2 - 0.135 = 0.$$

This gives for 0.01 probability and hence U = 2.326

$$0.902\,x_p - 4.59\,x_p + 5.365 = 0$$

whose roots are $x_{.05} = 1.876$ and $x_{.95} = 3.252$. These may be measured from U = 2.326 giving 0.926 and -.450 for the 90% confidence limits. The first value corresponds to the author's value 0.944. Thus the author's confidence intervals are too wide. They are also wider than the true intervals given by the non-central t distribution as a check of Jennett and Welche's example showed where the author's method for n = 25 and U = .842 gave 0.690 against 0.462 for the correct method. It is interesting to note that the author's method produces the same results as the median where non-central t becomes a t-distribution because U = O.

M. A. BENSON,[8] A.M. ASCE, and S. E. RANTZ,[9] A.M. ASCE.—Mr. Beard has discussed an important engineering problem of our time, the determining of flood frequency and magnitude relationships for design purposes. He has made a comprehensive study of flood-peak distribution and has demonstrated some interesting results.

Various aspects of Mr. Beard's papers seem worthy of comment. Perhaps the most important is his use of maximum mean-daily flows rather than instantaneous peaks. If 24-hour discharges were to be used, then maximum 24-hour discharges would have much more meaning than calendar-day discharges. However, it is the peaks rather than the 24-hour mean flows that cause the extreme flooding and must be designed against. Only for the largest streams or those seriously affected by storage are the two sufficiently close so that either one could be used. As drainage area decreases, the use of daily instead of peak flows becomes progressively more devoid of meaning. The following is a sampling of stations used by Mr. Beard, showing the difference between daily and momentary yearly maxima in the 1951 water year:

No.	River	Drainage area (sq mi)	1951 Water Year maximum (cfs)	
			daily	momentary
804	Waterman Cn. Cr., Calif.	5	4.4	34
7	M. Br. Westfield R., Mass.	53	2,380	8,320
307	Canaseraga Creek, N. Y.	153	1,700	6,150
104	Roanoke River, Va.	388	4,130	10,600
657	No. Llano River, Tex.	914	934	5,500
811	Kings River, Calif.	1,690	51,600	91,000

8. Hydr. Engr., Geological Survey, U. S. Dept. of the Interior, Washington, D.C.
9. Hydr. Engr., Geological Survey, U. S. Dept. of the Interior, San Francisco, Calif.

In 1936, Jarvis[10] tabulated data on momentary peaks, maximum 24-hour averages, and maximum calendar-day averages, showing the large differences and variability between them. At that time it was necessary to base flood analyses on mean-daily discharges, although it was recognized that "On a stream where the peak stage or flow varies materially from the average flow for the day and especially where life and property may be jeopardized by even a short peak that exceeds a given stage and corresponding flow, consideration of such a peak may be a critical factor in some problems." There is no longer any necessity or valid reason for using any data other than momentary peaks for flood-frequency analyses. For recent years the data are available in that form. For older records the data can be analyzed and converted to that form on the basis of present-day records. This has been done in special studies made by the U. S. Geological Survey and a general program of tabulating all past peaks in that form is even now being carried on actively.

Mr. Beard discusses only two alternatives for computing plotting position, one of which is the California method, in which the recurrence interval is computed as N/M, the other the Hazen method, where the recurrence interval is N/(M-1/2). In these formulas N is the number of years of record and M is the order number, starting with 1 as the highest. The latter formula, although in general use for many years, has been found objectionable by modern-day statisticians. By this method, for example, the highest flood in a 20-year record is computed to be a 40-year flood. This amounts to an artificial extension of the period of record by a statistical device. The objection to the California method, cited by Mr. Beard, is done away with by use of the formula (N+1)/M.[11, 12, 13, 14] This formula conforms closely to modern theories and is coming into increasingly wider use. Despite his present plea for Mr. Hazen's formula, apparently Mr. Beard's earlier method,[15] now seemingly discarded, found its way into his table 4. The reader must indeed be confused. The plotting positions shown in table 4 are not strictly according to either of the two methods enumerated, but give values falling somewhere between them.

In general, Mr. Beard fails to acknowledge the recent developments in the statistical theory of extreme values and its widespread application to flood discharges.

Mr. Beard says, "It is entirely possible that some of the regions are characterized by peculiarities of skew or kurtosis, even though the aggregate is not." This is a very important consideration. He concludes that the individual distributions for the various gaging stations are normal (logarithmically) because no persistence is shown to exist in the two 20-year periods for his computed coefficients of skew and kurtosis. This line of reasoning is questionable.

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10. Floods in the United States, Magnitude and Frequency, by C. S. Jarvis and others, WSP 771, U. S. Geological Survey, 1936, pp. 90-113.
11. Review of Flood-Frequency Methods, Final Report of the Subcommittee of the Joint Division Committee of Floods, ASCE Separate No. 110, p. 3.
12. Graphical Approach to Statistics, by C. J. Velz, reprint from Water & Sewage Works, p. 6.
13. Frequency Analysis of Hydrologic Data with Special Application to Rainfall Intensities, by Ven Te Chow, Bulletin Series No. 414, Univ. of Ill. Eng. Experiment Station, 1953, pp. 26, 27.
14. Statistical Theory of Extreme Values and Some Practical Applications, by E. J. Gumbel, Applied Mathematics Series 33, Nat. Bur. of Stds., 1954, pp. 13-15.
15. Statistical Analysis in Hydrology, by L. R. Beard, ASCE Trans., Vol. 108, 1943, p. 1110.

Moreover, it has been shown by Slade[3] (p. 426) that ". . . skewness is never a truly significant characteristic when the sample from which it is computed has less than about 140 items, and that it is quite meaningless to use this measure when there are 50 or fewer items". The coefficient of kurtosis, involving a moment of higher order, would require even a larger sample. The analysis of skewness and kurtosis from 20-year samples would seem to have no significance. Adoption of a fixed, analytically fitted function to flood data requires the assumption that individual stations should conform to this pattern. Although the broad generalized data on floods may follow the logarithmic-normal distribution, this has not been demonstrated for individual stations or for stations within particular regions. The evidence shown by flood-frequency analyses for particular regions is that other and varied patterns may prevail over fairly wide areas.

On general grounds, there is a serious objection to using rigid mathematical methods of analysis of hydrologic data. The presumption of distributions such as the log-normal or others involves procedures which are equivalent to throwing the data into a mill and grinding out an answer, disregarding the hydrologic significance of the individual data. For example, it is common for a short flood record to contain a flood of very high frequency. Computation of the mean or standard deviation of such a sample would be overwhelmingly influenced by the one large item. The deficiencies in the strict mathematical approach have been discussed very ably by Velz.[12]

Another consequence of assuming a theoretically fixed distribution, such as the log-normal, is that plotted data are then assumed to define a straight-line or other fixed graph. This graph is then commonly used to obtain the answer to expectancies far beyond the limits of the original data. Randell[16] some time ago pointed out many objections to the fixed mathematical approach, among them the fact that such methods ". . . although apt, on account of their mathematical definiteness, to convey a false impression of truth and accuracy, are practically valueless for estimates involving long periods".

Mr. Beard discusses the need for region-wide analyses and says that "a comprehensive, nation-wide analysis of flood flows will be required in order to establish adequate criteria and procedures for flood-frequency determinations". The U. S. Geological Survey has made studies of region-wide flood-frequency relationships in some states and is currently working on others. Results of such studies have already been published for Massachusetts, Minnesota, Georgia, Western Washington, Louisiana, and Iowa. In addition, studies have already been initiated on a nation-wide basis.

G. N. ALEXANDER,[17] A.M. ASCE.—The author's paper is a valuable contribution to the subject of flood probabilities, using recognized statistical techniques. However the writer is not convinced that he has proved his first conclusion that "with rare exceptions the logs of the annual maxima of mean daily flows are normally distributed". He states that this finding apparently "completes the accomplishment of this preliminary lack" of providing a "theoretically complete and adequately documented exposition of flood-flow distribution theory".

This would seem a rather pretentious claim as, for instance, no reference is made to any other theories of the distribution, such as the extreme-value

16. Discussion on Probability of Flood Flows, by Ralph R. Randell, Proc. ASCE, 1928, p. 205.
17. Designing Engr., Water Resources, State Rivers and Water Supply Comm., Victoria, Australia.

theory, which will give a different distribution from that proposed by the author. Also, a flood flow distribution theory should take some account of the hydrological factors, as indicated by Horton.(1) The fitting of curves must still be regarded as an empirical process and results can therefore only be judged by graphical or statistical tests. As graphical inspection can be misleading, and there are inherent difficulties in standard chi-square tests because of correlation between different streams, even the normal probability criteria are lacking.

The author's 'proof' of the general validity of the log-normal distribution largely hinges on the results obtained from his Step 4 illustrated in Fig. 2. He shows that the logs of annual maximum mean daily flows of all the stations are close to a normal distribution; how close is left to the reader to judge from inspection of the graph, no chi-square test being applied.

However, even if such a test indicated the log-normal was acceptable for the aggregate, this does not necessarily mean that the individual stations are characterized by the log-normal, and Step 5 is an attempt to examine this problem.

In this step the author has shown that there is no correlation between the values of skewness obtained for successive twenty-year periods of 45 streams. The validity of this test, or for that matter any other test based on computed values of skewness, is questionable owing to the relatively high standard error associated with parameters based on third moments, particularly for periods as short as twenty years.

As the author is trying to show that the distribution is log-normal, it is appropriate to test whether the mean value of SK given in Table 3 departs significantly from its expected value of zero. Although the t-test shows no significant difference, the high standard error of SK again largely nullifies the results of the test.

In order to carry out a test on a large number of streams the writer graphically correlated the values of the co-efficients of variation and skewness of the 194 streams of Western U. S. A. given in the U. S. B. R. Manual.(2) Chow(3) has recently shown that these coefficients are related for the log-normal distribution. However, his theoretical relationship departed markedly from the mean curve drawn through the plotted points, even after allowing for the slight differences in the formulae used in computing Cv and Cs in the U. S. B. R. Manual, and those used by Chow.

It would therefore be informative if such a test were applied to the streams given in Table 1. Until such time as more comprehensive tests such as these are carried out, it seems premature to assert the author's first conclusion that departures from the log-normal are rare. The log-normal appears to be the most useful of skew distributions for hydrologic analysis, but whether departures from it are characteristic or due to sampling errors needs further investigation.

It should be noted that the value "n" used in equations 5 and 6 are not the same. In Equation 6, "n" is the number of years, whereas in Equation 5 it is the number of degrees of freedom, which in general is the number of years less one.

REFERENCES

1. Horton 1936 U. S. G. S. Water Supply Paper No. 771. Floods in the United States.

2. U. S. Bureau of Reclamation Manual. Vol. IV. Chap. 6.4.

3. Chow. "The Log-Probability Law and Its Engineering Applications." (Unpublished as yet).

CORRECTIONS FOR TRANSACTIONS.—In Table 3, for Station 401, the two S-values should be 0.318 and 0.306 instead of 1.006 and 0.969, respectively.

On page 5, in the tenth line from the bottom, "Table 2" should be changed to read "Table 3".

On page 7, line 1, "Table 4" should be changed to read "Table 2".

Table 5, referred to but not included in the paper, is published herewith.

TABLE 5

ERRORS OF ESTIMATED VALUES

As Coefficients of Standard Deviation

Level of Significance (P)*	Years of Record (n)	Exceedence Frequency						
		.001	.010	.100	.500	.900	.990	.999
.05	5	4.35	3.34	2.00	.95	1.05	1.26	1.45
	10	2.08	1.61	1.01	.58	.68	.86	1.02
	15	1.49	1.16	.74	.46	.54	.70	.85
	20	1.21	.95	.61	.39	.47	.62	.74
	30	.92	.72	.48	.31	.38	.51	.62
	40	.78	.62	.40	.27	.33	.44	.54
	50	.68	.54	.36	.24	.30	.42	.52
	70	.56	.44	.29	.20	.25	.34	.42
	100	.45	.36	.24	.17	.21	.29	.36
.25	5	1.41	1.08	.66	.33	.38	.47	.55
	10	.76	.59	.38	.22	.27	.34	.41
	15	.57	.44	.29	.18	.22	.29	.35
	20	.47	.37	.24	.15	.19	.26	.31
	30	.36	.28	.19	.12	.16	.21	.26
	40	.30	.24	.16	.11	.14	.19	.23
	50	.26	.21	.14	.10	.12	.17	.21
	70	.22	.18	.12	.08	.10	.14	.18
	100	.18	.14	.10	.07	.09	.12	.15
.75	5	-.55	-.47	-.38	-.33	-.66	-1.08	-1.41
	10	-.41	-.34	-.27	-.22	-.38	-.59	-.76
	15	-.35	-.29	-.22	-.18	-.29	-.44	-.57
	20	-.31	-.26	-.19	-.15	-.24	-.37	-.47
	30	-.26	-.21	-.16	-.12	-.19	-.28	-.36
	40	-.23	-.19	-.14	-.11	-.16	-.24	-.30
	50	-.21	-.17	-.12	-.10	-.14	-.21	-.26
	70	-.18	-.14	-.10	-.08	-.12	-.18	-.22
	100	-.15	-.12	-.09	-.07	-.10	-.14	-.18
.95	5	-1.45	-1.26	-1.05	-.95	-2.00	-3.34	-4.35
	10	-1.02	-.86	-.68	-.58	-1.01	-1.61	-2.08
	15	-.85	-.70	-.54	-.46	-.74	-1.16	-1.49
	20	-.74	-.62	-.47	-.39	-.61	-.95	-1.21
	30	-.62	-.51	-.38	-.31	-.48	-.72	-.92
	40	-.54	-.44	-.33	-.27	-.40	-.62	-.78
	50	-.52	-.42	-.30	-.24	-.36	-.54	-.68
	70	-.42	-.34	-.25	-.20	-.29	-.44	-.56
	100	-.36	-.29	-.21	-.17	-.24	-.36	-.45

*Chance of True Value Being Greater than Sum of Estimated Value and Given Error.

Discussion of
"STATISTICAL THEORY OF DROUGHTS"

by E. J. Gumbel
(Proc. Sep. 439)

PERRY M. FORD,[1]—The author, in restricting the concept of drought to an adverse condition of 1-day duration, "droughts which are defined as the annual minima of discharges" (page 439-1), seriously limits the usefulness of the method. Furthermore, the season of the low flow is disregarded. This concept of drought is, of course, in keeping with the theory of extreme values.

Probably of much greater concern regarding droughts is the commoner concept—a continued lack of moisture. That the two concepts are mutually dissociated is illustrated by the data in the author's Table III, page 439-8. This tabulation lists 1,000 cfs as the minimum flow of the Colorado River at Lees Ferry, for the period 1922 through 1939. As shown in Geological Survey, Water Supply Paper 609, this minimum occurred on December 27, 1925. However, the water-year discharge for that year was 11,300,000 acre-feet. The relative magnitude of this 1925 discharge with respect to drought conditions becomes evident when compared with the 4,377,000-acre-foot discharge of 1934. During the 18-year period 1922 through 1939, there were 5 years in which the water-year discharge was less than that of 1925. The dissociation may be extended to characteristics of frequency distributions. That is, analyses have shown that frequency distributions for flows of short duration, as for 1 day, differ from those of longer duration.

Interesting paradoxical interpretations have come to the writer's attention regarding the meaning of asymptotic (page 439-1) as applied to frequency studies. To some readers the word brings to mind an accumulated frequency curve which approaches a limiting magnitude such as a maximum possible flow, and would indicate a limited function; whereas, this same word may be intended to describe a normal curve of error which is asymptotic to a zero base line, and in this sense would indicate an unlimited function. This is perhaps one of the many points that might be clarified by eminent authorities, as Dr. Gumbel, in the interest of a better understanding between statisticians and their readers.

In applying the theory of extreme values to streamflow, the author selects the highest (or lowest) of a large number, 365, of daily discharges. Since one is dealing with laws of chance, it would seem that one must assume that any one of the 365 events has an equal chance of being the highest (or lowest) of that number. Seasonal fluctuations in flow would preclude such assumption in many basins.

R. D. GOODRICH,[2] M. ASCE.—This paper is a very important contribution to the literature on the frequencies and probabilities of the occurrence of observational data in the fields of hydraulics and hydrology, since it supplies

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2. Chf. Engr., Upper Colorado River Comm., Grand Junction, Colo.

reliable methods for the investigation of data hitherto seldom studied statistically. In this discussion no attempt will be made to analyze the mathematics of the theory presented in the paper but only to call attention to the similarity between some of the formulas and quantities in this paper and one published in the Transactions of the Society some years ago.[3]

In the paper referred to a general equation is given and a special plotting paper is described upon which several types of skew frequency distributions can be so modified as to be plotted approximately along straight lines. Extrapolation can then be carried out to a limited extent at either the upper or lower end of the curve.

In the author's equation the values of the median and the mode are equal when his factor 1/ = 0.30685(2.4), while for the writer's equation the mean and mode are equal when his factor 1/c = 0.30192.

A more striking agreement between these two papers is shown by the fact that the author's criterian, $\bar{x}/s$ in terms of the gamma function, to tell "whether or not zero should be chosen as the lower limit." Formula (4.9) is the reciprocal of the writer's formula for the coefficient of variation for his equation when the lower limit is zero. /Formula (7) page 8 of Footnote (1)./

The writer's equation is very general, and includes a form with the lower limit greater than zero in which case an asymptote to the curve may be used to define the value of the lower limit. While the writer had to depend largely upon an approximate graphical determination for his lower limit and hence for the location of the asymptote, the author now supplies a more reliable method by which the probability of the occurrence of the estimated smallest value can be determined and it is thought that this method may also be applicable for use with the writer's earlier equations to determine a suitable lower limit.

Another very valuable contribution by Mr. Gumbel is his analysis of the character and use of the "third moment" required in fitting data of skewed frequency distributions to any one of the several equations now in general use if the method of moments is to be used. The writer has been using his equations with considerable success by first making the logarithmic transformation and then fitting a curve to the data by the method of least squares. This in effect substitutes for the arithmetic mean used in a normal distribution, the geometric mean of the observations which is the antilog of the mean of the series of logarithms. Then the summation of the squares of the departures of the logarithmic series from their mean together with the summation of the products of the departures of both the independent and dependent transformed variates from their respective means are used in place of the second and third moments in the process of curve fitting. Certainly the method of least squares is well known and widely used and formulas for the standard error of estimate, of the mean, and of the regression coefficient are readily available. It might be noted in passing that while Mr. Gumbel's method for floods assumes a single fixed skewness for all distributions, the writer's method and equations can be adapted to almost any degree of skewness, whether positive or negative.

The writer has used his methods for nearly thirty years, and he will continue to use them for either floods or droughts with even more confidence than ever when skew frequency hydraulic data is to be investigated.

The author should be commended for his able presentation of a highly technical and difficult subject and it is a source of some satisfaction to this writer to learn that his own exponential equation appears to have much greater justification from a mathematical standpoint than was previously thought possible.

3. Straight line Plotting of Skew Frequency Data, by R. D. Goodrich, Trans. ASCE Vol. 91 December 1927, page 1.

SHUH-CHAI LEE.[4]—After his contribution to the statistical theory of floods,[5] Dr. E. J. Gumbel has recently presented this paper on droughts. In statistical meaning, floods and droughts both are extreme values. But the unlimited distribution of the exponential type that was originally used for floods and that since been used for droughts[6] is not appropriate as mentioned by the author. Therefore Mr. Gumbel recommends the method to make use of the asymptotic theory of smallest values of a limited statistical variate.

This procedure has distinct advantages: (1) The extremal probability paper designed for floods can also be used for the logarithms of droughts; (2) If the lower limit of the discharges is assumed to be zero the probability function of the droughts becomes a straight line; (3) The slope of the line is independent of the measures chosen for the droughts.

The first and second points are welcome to engineers because of easier computation and extrapolation. Besides, the famous Student's t-distribution can be used to test the significance of the regression coefficient or, in engineering terms, in determining the accuracy of the slope, say $b = 1/\alpha'$, of the line.[7] For the existing number N of observed droughts, the accuracy e of the slope of the line is

$$e = \pm 100(1-\beta/b)\% = \frac{t}{\sqrt{N}} \frac{s(\log X)}{b\sigma_{\hat{N}}}$$

where stands for the ideal value of b,

$$s(\log X) = \sqrt{\frac{\Sigma(\log x - \log X)^2}{N-2}}$$

the mean square deviation from the line of regression, logX=logu-by, or the standard error of estimate of logx,

$$\sigma_N = \sqrt{\frac{\Sigma(y - \bar{y}_N)^2}{N}}$$

the standard deviation of y from $\bar{y}_N$, and the Student's t-value corresponding to (N-2) degrees of freedom and a desired probability may be found in any textbook of statistics. Consequently, the equation of the line with confidence intervals written as

$$\log X = \log u - b(1 \pm e)y$$

can be used instead of control curves[8] for simplification.

4. National Taiwan University, Taipei, Taiwan, China.
5. "The Return Period of Flood Flows," by E. J. Gumbel, Ann. Math. Statistics, Vol. 12, No. 2, pp. 163-190, June 1941.
6. "Graphical Approach to Statistics," by C. J. Velz, Water and Sewage Works, Vol. 97, No. 10, New York, 1950.
7. "Length of Record and Number of Stations Required in Statistical Analysis of Hydrologic Data," presented by Shuh-Chai Lee at UNECAFE Regional Technical Conference on Water Resources Development, Tokyo, Japan, 17-22 May, 1954.
8. "Statistical Control Curves for Flood Discharges," by E. J. Gumbel, Tran. Am. Geophys. Union, Part II, pp. 489-509, Nov. 1942.

For the third point, it plays the same part as the method used in the writer's previous paper,[9] that converts each item of flow data into ratio to the mean. That procedure is convenient to show variations of the slope of the line in relation to the river characteristics.

The above two points lose their effect as the relation between the reduced variate y and logx is no longer linear if the lower limit of droughts exceeds zero. Is there another measure which can bridge this gap?

9. "The Return-period and Variability of Floods in China," by Shuh-Chai Lee, Essays and Papers in Memory of Late President Fu Ssu-Nien, National Taiwan Univ., Taiwan, China, pp. 471-496, 1952.

Discussion of
"SURFACE-WATER SUPPLY FOR IRRIGATION IN THE VERMILION RIVER BASIN, LOUISIANA"

by E. L. Hendricks
(Proc. Sep. 489)

C. K. OAKES,[1] M. ASCE.—Mr. Hendricks is to be highly commended for the collecting, compiling and analyses of the data on the needs and supplies of water for irrigation purposes throughout the Teche-Vermilion system. For the first time data even approaching sufficiency in scope and quantities to permit such an analyses have been obtained. These data will form the basis for the projection of final plans and the design of facilities to alleviate the fresh-water shortage that has frequently occurred during the past. The data presented as to acreages and water requirements are indispensable to proper planning and designing to meet the needs imposed for irrigation purposes. The paper concerns only the Vermilion Basin. Since the entire program of the water resources investigation, by the U. S. G. S. of Southwest Louisiana covered the Vermilion, the Mermentau and the Calcasieu Basins, a brief statement regarding the discrepancies in information contained in various literature during the past is considered appropriate in this discussion.

Estimates made by different individuals during the past have set the amount required on the field at various depths ranging between 32 inches and about 48 inches. Estimates of that portion of the seasonal rainfall retained on the fields ranged from 5 inches to over 12 inches irrespective of the total seasonal rainfall. Estimates of the portion irrigated with ground-water ranged from 30% to 60% of the total rice acreage. Thus, for a rice crop of 600,000 acres (the total acreage annually planted to rice ranges between about 580,000 and about 620,000 acres) the total seasonal volume needed for irrigation was somewhere between 1,000,000 and 2,150,000 acre-feet of which the volume pumped from wells was between 300,000 and 1,300,000 acre-feet. Should a "middle-of-the-road" figure be adopted as to the total water requirements, one risked being in error by some half-million acre-feet for the seasonal demands.

For the Vermilion Basin the data presented by Mr. Hendricks and that obtained by the Ground-Water Branch of the Survey have reduced the 30% range in discrepancy of that portion using ground-water to about 5%. A comparison of the data appears to furnish most of the answer as to why the range of some 16 inches in the previously stated needs. From these data it is apparent that losses of about 35% between the pumping plants on the streams and the fields account for most or all of this 16-inch discrepancy.

A comparison of the quantities measured on the fields using surface water and that measured at the plants pumping ground-water, in conjunction with Mr. Hendricks' tabulation of the average total of 31.7 inches actually required on the field each season indicates that losses by downward seepage through the strata underlying the surface stratum is very small. Then the conveyance

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losses of about 35% should be mostly evaporation, lateral seepage through the delivery channel dykes and crevasses in these dykes. If these total losses of about 35% are of such nature that corrective measures cannot be effected within reasonable cost then the design of facilities to provide an adequate and dependable supply of fresh surface-water will necessarily be based upon the gross or pumpage requirements rather than upon the net or field requirements. It also means that if areas that presently use only ground-water should switch to surface-water as a source of supply then their gross needs will be about 50% more than their ground-water requirements. Thus should the area in the Vermilion Basin that annually uses about 32,000 acre-feet of ground-water be irrigated with surface-water, the quantity diverted from the river would be increased by about 48,000 acre-feet annually.

The author's statements relative to the entry of salt water to the system by way of Southwest Pass recalls to mind certain questions relative to the merits of closing this Pass. Although this issue was outside the scope of the investigation, it has been a controversial one for several years and therefore a few questions relative thereto are considered appropriate in this discussion.

If the Pass were closed, what would be the affect on this closure on the passage of Atchafalaya River water westward into West Cote Blanche and Vermilion Bays? What relationship does this movement of Atchafalaya River water bear to the winds, the tides and the pass serving as an escape valve from the bays to the Gulf? What compensation, if any, is effected by Southeast winds for the adversities of salt water passage through the Pass to the bay caused by Southwest winds?

Regarding the effects of closing Southwest Pass on the entry of waters from the Gulf observed conditions in Lake Pontchartrain are cited as what appear to be what might occur in Vermilion Bay.

Lake Pontchartrain has a volume of nearly 5 million acre-feet. This is nearly 3 times the combined volumes of the two bays in question. There are no diversions of consequence from the lake or its tributaries comparable to those from the Vermilion River and the storm runoff and low flow to the lake is better than that to Vermilion Bay and West Cote Blanche Bay. The clorine in Lake Pontchartrain at Frenier has been as much as 3000 ppm for periods of 3 months duration.

A comparison of channel dimensions shows that water entering Lake Pontchartrain through Chef Menteur and the Rigolets should meet greater resistance than that entering Vermilion Bay through Southwest Pass or East and West Cote Blanche Bays.

The pertinent and vital questions that arise in connection with the issue of closing Southwest Pass cannot be answered with any degree of reliability without adequate data covering the entire area embracing the coastal region from Vermilion Bay to Atchafalaya Bay. These data would include Atchafalaya River discharge rates, continuous records of salinity and of tides and currents in Southwest Pass and in the whole coastal region both as to directions, magnitudes and durations. Even then the effects of closing the pass on currents in this region would not be subject to a satisfactory analyses except by means of a model study.

In the author's accounting of the origin of the 178,000 acre-feet diverted from the Vermilion River during the period April 30 to August 12, 1948, no mention was made of ground-water inflow during the first half of the period. On the basis of the findings in Mr. Jones' investigations of the ground-water phase of the program it is estimated that this flow probably didn't exceed about 8,000 acre-feet. Also of the 37,000 acre-feet of "Vermilion Bay water

made usable by dilution" it appears likely that this dilution of the Bay's water was composed of both Atchafalaya River water and pre-seasonal runoff from the Vermilion River. Then on this basis with Gulf water having a concentration of 27,000 p.p.m. and that in Vermilion Bay having from 2,000 to 2,500 p.p.m. approximately 34,000 of the subject 37,000 acre-feet of water was composed of waters from both the Atchafalaya and Vermilion Rivers.

Thus the total pre-seasonal runoff from the Vermilion River that was reclaimed was somewhere between the limits of about 60,000 acre-feet and 68,000 acre-feet plus a certain indeterminate portion of the above mentioned 34,000 acre-feet. The runoff to the Bay during the 2-month period prior to April 30 was computed to be about 169,000 acre-feet. It is notable that the data presented showed that the usable fresh-water supply that was re-claimed from the bay had a rather rapid or abrupt ending which terminated on or about June 24. In spite of the increase in the accumulated deficiency subsequent to this date and the resulting upstream flow from the Bay, little or no pre-seasonal runoff was in evidence. It appears likely that the quantity of pre-seasonal runoff that can be reclaimed may also be limited by the capacity of a certain portion of the Bay as well as the volume of runoff during a certain period preceeding the irrigating season as suggested by the author. The hachured portion of Vermilion Bay has a capacity of about 70,000 acre-feet. (The author determined the reclaimed pre-seasonal runoff to be some 69,000 acre-feet.)

If this portion is the limit of pre-seasonal runoff that can be reclaimed, then the volume of reclaimable runoff will be limited to about 70,000 acre-feet irrespective of the volume of runoff during the pre-seasonal 4-week period. If, on the other hand, the time limit governs, then the volume will be a variable quantity from year to year and it could be appreciably increased by diversions to the Vermilion River through Ruth Canal and Bayou Fusilier for a certain period preceeding the irrigating season.

However, insofar as the plans and design of facilities for fresh-water supplies for the Vermilion Basin are concerned, this phase of the investigation will be more academic than of practicable use for the following reasons: (1) The uncertainty of pre-seasonal runoff as a dependable supply; (2) the most effective method of operating the Mermentau Basin System will likely draw heavily from the Vermilion Bay; and (3) the indications that an adequate and dependable supply of fresh water can be provided and will be necessary for the Vermilion system by the construction of a reservoir on the Lake Cocodrie area.

The author's statement that "The investigation conducted in the Vermilion River Basin indicates clearly that to prevent contamination of the ground-water aquifer the encroachment of salt-water in Vermilion River channel must not be allowed at any time" was undoubtedly based on the findings by the Ground-Water Branch of the Survey since evaluation of losses from the river to the aquifer was not made in the surface-water phase of the investigation. The statement which follows that "This would also insure a fresh-water supply to all irrigators in the basin" omits the explanation that this is contingent upon making available more usable fresh-water by either one or a combination of the following: (1) diversions from sources other than those presently drawn upon; (2) by storage in reservoirs on the headwaters; or (3) by increasing the amount of usable fresh water in Vermilion Bay.

The findings that salinity intrusions into Vermilion River channel occur as a result of diversions exceeding the headwater supplies and not because of the difference in density between the fresh-water supply in the river and that

which enters the channel from the bay has a very significant bearing on the design of facilities to provide fresh surface-water. Very little flow will be required downstream from the lowermost point of diversion on the river or from the lowermost point where recharging of the aquifer from the channel's bed can occur.

Therefore, the design can be based upon the present and future needs for diversion with an allowance for increased losses as recharge to the aquifer from the Vermilion River channel.